ASSESSMENT OF LANDFILL LEACHATE TREATMENT OPTIONS FOR WESTMORLAND/ALBERT REGIONAL SOLID WASTE COMMISSION

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1.0 SUMMARY

All landfills generate leachate which must be treated before being released to the environment. The degree of treatment is dependent on whether or not discharge to a municipal wastewater treatment system is viable. If a municipal treatment plant is available, only pretreatment of the leachate is required; if no municipal plant is available, then complete treatment is necessary.

Leachate is a complex wastewater whose characteristics, including flow, are difficult to predict. It is said that the only constant parameter of leachate is its variability.

This report discusses the major factors that affect leachate quality and flow. The Hydrologic Evaluation of Landfill Performance (HELP) model was used to assess leachate flow for different landfill management practices.

Based on estimates of leachate flow and organic strength, preliminary design and costing is presented. Estimates of capital and annual costs for the leachate treatment systems are presented in Table 1.1.

TABLE 1.1
CAPITAL AND ANNUAL COSTS
FOR LEACHATE TREATMENT SYSTEMS

CAPPING STRATEGY	TREATMENT SYSTEM TYPE	CAPITAL COST	ANNUAL OPERATING COST
Cells Capped After 5 Years	Pretreatment	$ 640,000	$ 72,000
	Complete Treatment	$4,260,000	$268,000
Cells Capped After 25 Years	Pretreatment	$1,230,000	$121,000
	Complete Treatment	$5,980,000	$529,000

2.0 INTRODUCTION

2.1 Objectives

The objectives of this report are to:

- review landfill leachate characteristics;
- summarize landfill leachate treatment and management options;
- estimate leachate flows and composition for the Westmorland/Albert Regional Sanitary Landfill; and
- prepare preliminary cost estimates for appropriate leachate treatment systems.

2.2 Landfill Leachate

Landfill leachate is produced when water, mainly from precipitation, percolates down through the compacted refuse in a sanitary landfill. The leachate contains a wide variety of organic and inorganic contaminants, due to the ongoing chemical and biological reactions within the landfill.

Leachate flow and composition vary considerably from landfill to landfill because of the following factors:

- age of the landfill;
- location of the landfill;
- refuse contents of the landfill; and

- landfill management practices (i.e., capping, recycle, shredding, baling, etc.).

It is important to recognize that leachate characteristics change as time passes at a given landfill due to the stabilization of the landfill. The landfill stabilization process may be broken into five phases, as follows:

Phase I - Initial

- Initial waste placement and moisture accumulation.
- Subsidence and closure of each landfill area.
- Onset of biological stabilization process.

Phase II - Transition

- Field capacity is approached and leachate is formed.
- A transition from initial aerobic to facultative and anaerobic microbial stabilization occurs.
- The primary electron acceptor shifts from oxygen to nitrates and sulfates with the displacement of oxygen by carbon dioxide in the gas.
- A trend toward reducing conditions is established.
- Measurable intermediates such as volatile fatty acids first appear in the leachate.

Phase III - Acid Formation

- Volatile acids become predominant with the continuing hydrolysis and fermentation of waste and leachate constituents.
- A decrease in pH occurs with mobilization and possible complexation of metal species.
- Nutrients such as nitrogen and phosphorus are released and partially utilized in support of the growth of biomass.

Phase IV - Methane Production

- Volatile acids appearing during the acid formation phase are converted to methane and carbon dioxide.
- The pH, formerly kept low by the volatile acids, returns to near neutrality and is buffered by the bicarbonate buffering system.
- Oxidation-reduction potentials (ORP) are at their lowest values, sulfates and nitrates are reduced to sulfides and ammonia.
- Nutrients continue to be consumed.
- Complexation and precipitation of metal species with sulfides and organic ligands.
- Leachate organic strength is decreased in correspondence with increases in gas production.

Phase V - Final

- Relative dormancy following active biological stabilization of the readily available organic constituents in the waste and leachate.

- Nutrients may become limiting.
- Measurable gas production all but ceases.
- Natural environmental conditions become reinstated.
- Oxygen and oxidized species may slowly reappear with a corresponding increase in oxidation-reduction potential.
- More microbially resistant organic materials may be slowly converted with the possible production of humic-like substances capable of complexing with and re-mobilizing heavy metals.

Naturally, as a large landfill site is developed, different parts of the landfill will be at different stages in the stabilization process. Therefore, the leachate collected for treatment will usually reflect characteristics based upon a mixture of the stabilization phases.

The changing characteristics of landfill leachate, due to the landfill stabilization, are demonstrated in Table 2.1. Table 2.2 presents a data summary for a variety of landfill leachate sources, which further illustrates the variability of leachate composition.

TABLE 2.1 - LEACHATE CHARACTERISTICS

PARAMETER	LEACHATE FROM RECENT WASTES	LEACHATE FROM AGED WASTES
pH	6.2	7.5
COD (Chemical Oxygen Demand)	23800	1160
BOD (Biochemical Oxygen Demand)	11900	260
TOC (Total Organic Carbon)	8000	465
Volatile Fatty Acids (as C)	5688	5
NH_3-N	790	370
NO_3-N+$N0_2$-N	3	1
Phosphate (PO_4)	0.73	1.4
Chloride (CI)	1315	2080
Sodium (Na)	960	1300
Magnesium (Mg)	252	185
Potassium (K)	780	590
Calcium (Ca)	1820	250
Manganese (Mn)	27	2.1
Iron (Fe)	540	23
Nickel (Ni)	0.6	0.1
Copper (Cu)	0.12	0.3
Zinc (Zn)	21.5	0.4
Lead (Pb)	1.4	0.14
All Values except pH in mg/L		

TABLE 2.2
RANGE OF LEACHATE COMPOSITION

PARAMETER	RANGE OF VALUES
COD	40 - 80,000
BOD	81 - 33,000
TOC	256 - 28,000
pH	3.7 - 8.5
TS	0 - 59,000
TDS	584 - 44,900
SS	10 - 700
Conductivity	2,810 - 16,800
Alkalinity ($CaC0_3$)	0 - 20,850
Hardness ($CaC0_3$)	0 - 22,800
Total P	0 - 130
Ortho P	6.5 - 85
NH_3-N	0 - 1,106
$N0_3$-N+$N0_2$-N	0.2 - 10.29
Ca	60 - 7,200
Cl	4.7 - 2,467
Na	0 - 7,700
K	28 - 3,770
$S0_4$	1 - 1,558
Mn	0.09 - 125
Mg	17 - 15,600
Fe	0 - 2,820
Zn	0 - 370
Cu	0 - 9.9
Cd	0.03 - 17
Pb	<0.10 - 2.0

Data compiled by Chian and DeWalle
(All values except pH and Conductivity in mg/L (Conductivity in μmhos/cm)

3.0 TREATMENT OF LANDFILL LEACHATE

3.1 Introduction

In general, the options for treatment of landfill leachate may be summarized as follows:

- site attenuation;
- discharge to municipal wastewater treatment plant (WWTP), with or without on-site pretreatment; and
- full, on-site secondary treatment for discharge of treated effluent to a watercourse.

The site attenuation option basically uses the microbiological community of the soil around the landfill to provide for treatment of the landfill leachate. Naturally, this option requires that the landfill be located in an area where soil and groundwater conditions are suitable. Under the Province of New Brunswick's guidelines for regional sanitary landfills, the site attenuation option is not acceptable. The guidelines specify that leachate is to be collected and treated.

Discharge of leachate to the municipal WWTP, when available, is the preferred treatment option. This approach can provide the lowest overall cost for leachate management because a new treatment plant is not required, although operating costs at the existing municipal WWTP would increase due to the increased load to the plant from the leachate. Direct discharge of landfill leachate to the municipal WWTP is possible when:

- the municipal WWTP is a secondary treatment plant with enough capacity to handle the leachate load;

- the leachate does not unduly stress or inhibit operation of the municipal WWTP.

As a general guideline, as long as leachate flow does not represent greater than five percent of the total waste flow at a municipal WWTP, then there should be no problems. Of course, each situation must be examined in more detail in order to assess the potential impacts on the municipal WWTP. If the municipal WWTP is a primary treatment plant, such as the new Moncton plant, then the leachate should be aerobically pretreated, on-site, to reduce BOD.

The third option, to provide an on-site, dedicated, secondary treatment plant for the leachate, is relatively expensive to build and operate. This option does, however, allow direct discharge of the treated effluent to a nearby watercourse. For comparison purposes, the costs of building and operating the on-site WWTP must be compared to the costs to deliver leachate to the municipal WWTP. Included in the cost to discharge to a municipal WWTP should be any cost associated with on-site pretreatment.

3.2 Leachate Treatment Options

Typically, landfill leachate represents a difficult waste stream to treat because of the following characteristics:

- high strength and variability (i.e., COD of 40 to 80,000 mg/L);

- low phosphorus concentrations;
- relatively high metals concentrations;
- excess nitrogen available (i.e., nitrogen removal may be required);
- flow may vary considerably, depending upon precipitation; and
- characteristics change as landfill ages.

In addition, sanitary landfills are seldom located close to a municipal WWTP so it can be costly to transport the leachate to the nearest WWTP.

Specific technologies which would be applicable for the Westmorland/Albert sanitary landfill leachate include the following:

- aerobic pretreatment with effluent discharged to Moncton regional WWTP;
- physical/chemical treatment;
- anaerobic biological treatment; and
- secondary aerobic treatment.

The objective of the aerobic pretreatment option would be to reduce the impact of the leachate on the Moncton WWTP by removing BOD and reducing metals concentrations (through precipitation). The aerobically pretreated effluent would not be suitable for direct discharge to a nearby watercourse but would be piped or trucked to the Moncton regional WWTP (depending upon landfill location).

Although there would be many options for aerobic pretreatment, the most cost effective, reliable and simple to operate would basically consist of a multi-cell, long retention time (i.e., 40 days) aerated lagoon. The lagoon would be designed as a multi-cell unit to maximize BOD removal efficiency. It is possible to further

improve aerated lagoon efficiency by covering some, or all, of the aerated cells to minimize heat loss during cold weather. ADI Limited has a patent pending on a covered aerobic reactor, designated as the ADI-CAR system.

Physical/chemical treatment for sanitary landfill leachate could consist of:

- precipitation/coagulation;
- oxidation; or
- activated carbon.

It may be necessary to use physical/chemical treatment to reduce metals concentrations prior to biological treatment or to polish biologically treated effluent. If the objective of physical/chemical treatment is to reduce metals concentrations, then precipitation/coagulation is a logical and cost-effective choice. The complete precipitation/coagulation system would consist of the following:

- chemical addition system to raise leachate pH to precipitate dissolved metals, complete with chemical storage (i.e., lime, caustic soda, magnesium hydroxide) and feeding system, mixed reaction vessel and pH control system;

- chemical addition system to coagulate precipitated particles (i.e., ferric sulfate, etc.); and

- clarifier to remove precipitated metals as a sludge (which would be wasted to the landfill).

Wastewaters which have a very high concentration of organic matter (like leachate) can be difficult and expensive to treat biologically in conventional aerobic treatment systems. Anaerobic biological treatment systems are well suited for high strength wastes and have the following advantages over aerobic systems:

- generate much less waste sludge;
- produce biogas which may be used as a fuel; and
- no aeration used, so electrical energy demand reduced.

However, anaerobic reactors do not operate efficiently at very low temperatures (i.e., less than 15°C). Low-rate anaerobic reactors (such as the ADI-BVF®) normally require a temperature of about 20°C, while high-rate reactors (such as the upflow anaerobic sludge blanket) normally operate at 30-35°C. Therefore, leachate would need to be preheated to ensure efficient anaerobic reactor operation. The increased temperature will also benefit the subsequent aerobic polishing step. The biogas generated by the anaerobic process could be used to produce some of the heat necessary to preheat the leachate for anaerobic treatment. If landfill gas was collected, then, it could also provide the energy to preheat leachate.

Following the anaerobic reactor, an aerobic polishing step would be necessary to further reduce BOD and solids before the treated leachate could be discharged to a water course. The sequencing batch reactor (SBR), a modification of the activated sludge process, would be well suited for this application due to its flexibility of operation and ability to remove nitrogen from the waste stream.

The processes necessary to operate an SBR successfully have been known since the early 1900s. The forerunner of the SBR and other activated sludge processes was a fill-and-draw system operated as a batch process. However, due to the high degree of manual operations necessary for successful operation, fill-and-draw systems were not used to any extent. With today's control technology, this is no longer the case.

An SBR can be used to treat any wastewater that can be treated by an activated sludge system. Some research suggests that an SBR can treat wastewaters that conventional activated sludge cannot. The SBR system has the ability to remove BOD, nitrogen (nitrification and denitrification) and phosphorus by changing operational procedures.

In its simplest form, the batch reactor consists of a single vessel where timed processes take place sequentially (Figure 3.1). The process may be controlled by level sensors, DO probes and turbidity sensors. The liquid level cycles between the low water level and the high water level. An SBR passes through four or five phases which are FILL, REACT, SETTLE, DECANT and, in some cases, IDLE. During a typical cycle, the following takes place:

FILL — During FILL, the reactor is mixed but not necessarily aerated. The influent mingles with the mixed liquor in the reactor for solids/liquid contact and consequent uptake of soluble BOD.

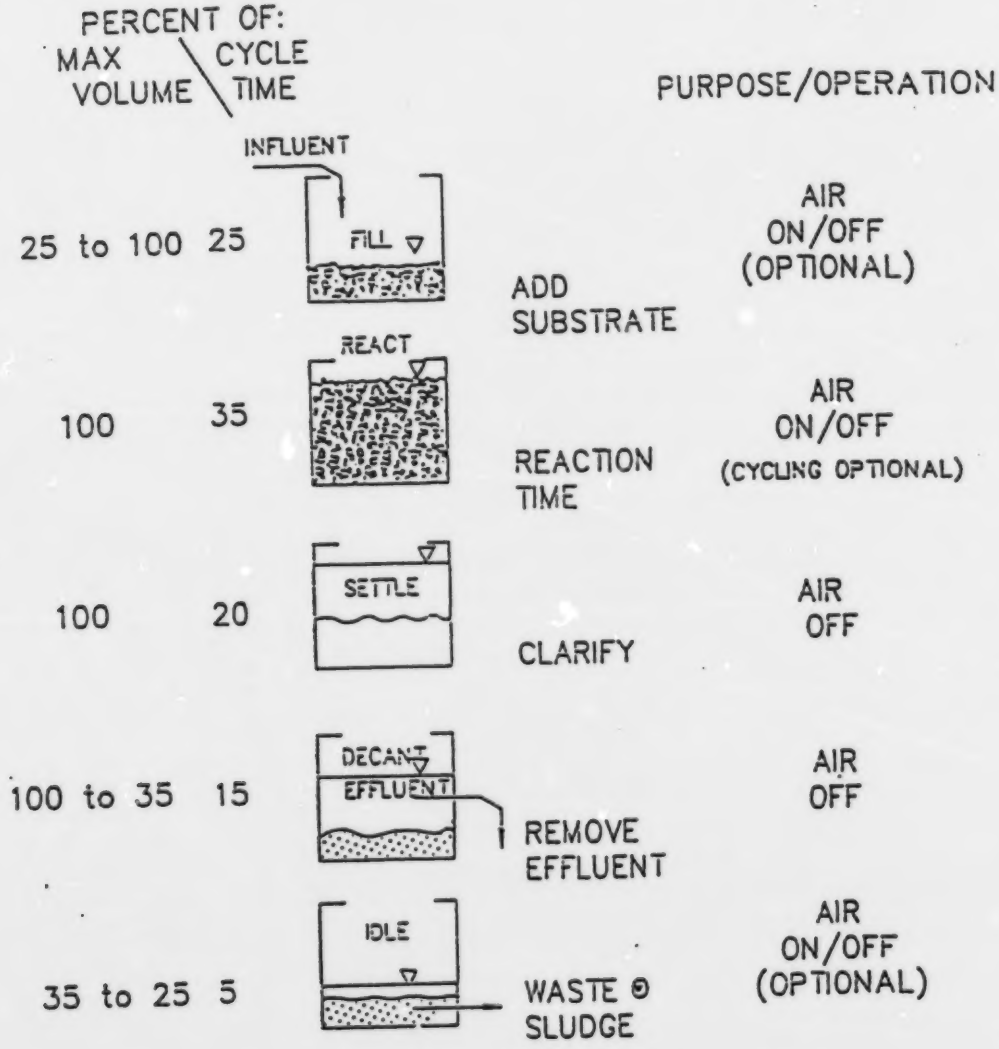

Figure 3.1 - Typical SBR Operation

REACT Aeration commences (if it has not already done so during FILL) and the majority of the biochemical reactions take place. Mixing of the reactor contents continues; however, aeration can be cycled on and off to promote nitrification and denitrification.

SETTLE Mixing and aeration are stopped, and the biological solids are allowed to settle.

DECANT Clarified liquor is decanted.

IDLE If there is a multiple-tank system, the tank may wait for the next charge of influent. Also, the idle period can be used to waste sludge.

The single-tank SBR system is applicable for noncontinuous flow situations, such as those that occur in the food-processing industry; continuous, higher flow situations require a multi-tank system. Minimal operator input is required, since SBR control functions are executed automatically.

Certainly, an SBR would not be the only alternative for polishing the anaerobically pretreated leachate but an SBR has some advantages over conventional activated sludge including:

- variations in wastewater loadings (hydraulic or organic) are more easily accommodated due to the inherent flexibility of the SBR (ability to vary cycle times is somewhat like having the ability to increase/decrease the size of aeration basins or clarifiers);

- SBR may be more resistant to shock loadings due to large sludge mass (usually more than conventional activated sludge);

- anoxic periods can be incorporated to promote denitrification and enhance nitrogen removal, control growth of filamentous organisms and reduce power consumption (increased oxygen transfer after anoxic periods due to increased dissolution driving force);

- floc-forming organisms are favoured over filamentous organisms (filamentous organisms cause the sludge to "bulk" which can lead to solids carryover in the effluent) due to a high substrate tension, or the abundance of food per unit of biomass present during the initial part of the FILL cycle; higher populations floc-forming organisms make for better settling sludge;

- an SBR restarts more easily after shutdown because the organisms are acclimated to feed-starve cycles;

- the surface area for liquid-solids separation in the SBRs is generally five to ten times that of conventionally designed secondary clarifiers; liquid-solids separation takes place during near-ideal, quiescent conditions (i.e., no inflow/outflow); and

- the capital cost is reduced due to the elimination of secondary clarifiers and sludge pumping stations.

For treatment of sanitary landfill leachate, a pilot-plant study should precede full-scale design to confirm design criteria and treatment plant performance.

3.3 Landfill Operating Factors

There are a number of possible operating factors at a sanitary landfill which can have very significant effects on leachate quantity, composition and treatment, including:

- leachate recycle to the landfill;
- shredding of refuse;
- baling of refuse;
- compaction density;
- types of waste allowed;
- addition of sewage sludge to the landfill; and
- capping frequency/strategy at the landfill.

The recycle of leachate to the landfill has been used as a leachate treatment strategy for some time. Various studies have shown that the landfill can be used as an anaerobic roughing filter to reduce leachate COD. Some of the advantages of leachate recycle are:

- provides some treatment for leachate;
- can reduce variability of leachate composition; and
- landfill may be stabilized and converted to another use more quickly.

Shredding and baling are two refuse processing options which may be used to increase the insitu density of the solid waste. Shredding decreases particle size and therefore increases landfill capacity but decreases permeability of the refuse. The

water percolating through the refuse, therefore, spends more time in contact with smaller refuse particles, thereby increasing leachate strength. Baling, on the other hand, tends to decrease leachate strength because water percolates more easily around, rather than through, the dense bales. This is particularly true if the refuse is shredded prior to baling.

Compaction will provide greater use of the landfill volume. It should also have the effect of decreasing void space in the landfill and, thereby, decreasing field capacity. In this respect, leachate generation may start earlier in the life of the compacted cell. However, compaction also has the effect of allowing more refuse per unit volume, refuse which will absorb a certain quantity of water. Therefore, compaction should have little overall impact on leachate quantity.

There is little in the literature on quantifying quality changes due to shredding, compaction or baling. Their main function is increasing the storage capacity of the landfill. Naturally, leachate characteristics are affected by the wastes which are placed in the landfill. In order to minimize detrimental effects certain waste materials (i.e., pesticides, etc.) should not be permitted, but rather disposed of in a controlled location.

Addition of stabilized sewage sludge can be of some potential advantage to landfill operation by seeding the biological degradation process. In addition, the excess lime in a lime stabilized sludge (such as from the Moncton regional WWTP) could raise the pH in portions of the landfill thereby reducing metals concentrations in leachate. As a point of interest, the same benefit may be obtained by incorporating lime with the daily cover at the landfill. With a lime stabilized sludge, any potential for negative impacts (such as increased pathogens) would be

minimized. Furthermore, since leachate will be collected and treated, any effects of sewage sludge disposal on leachate will not be felt in the immediate environment (i.e., groundwater).

There are other properties of municipal sewage sludge that should be considered. The sludge will be 70 - 80 percent water which, when mixed with municipal waste, will increase the initial water content of the combined waste. Also, municipal sludge generally has moderately high concentrations of metal ions.

The metals in lime stabalized municipal sludge will be tightly bound and therefore slow to leach out. In combination with the leachable metal found in other components of municipal waste, the metal contribution of the sludge is not projected to be significant.

Centrifuged municipal primary sludge at 25 percent solids will have a density just above 1000 kg/m^3. If all municipal sludge is to be landfilled, it will have an impact on both the lifetime of the landfill and how fast the landfill reaches field capacity. There should, however, be no resultant increase in the average daily leachate production.

The landfill capping strategy has a tremendous influence on both leachate flow and the rate of landfill stabilization. One possibility is to minimize leachate flow by capping each cell as soon as it is completed. Since this naturally reduces the rate of water infiltration into the refuse then the landfill stabilization process takes longer. On the other hand, it is possible to promote more rapid stabilization by not capping the landfill until the stabilization process is virtually complete. This strategy results in a much larger volume of leachate to treat. A compromise

between these extremes may be reached by capping completed cells and recycling leachate to promote more rapid stabilization.

3.4 Leachate Treatment Plant Commissioning

The proper design of any treatment works requires information on wastewater flows and organic loadings, and leachate treatment systems are no exception. Although model outputs are useful for predictive purposes and in preliminary design, final treatment system design should be based on actual field data, if available. The obvious problem is how to deal with the leachate until the treatment plant is designed and constructed.

The usual methods of pre-leachate treatment system leachate management are:

- trucking off-site; and
- containment.

As leachate flows from new cells are minimal until field capacity is attained, either method is feasible. However, the time-frame for data collection and system design and construction is tight. For the capping cases presented in this report, the Years 4, 5, and 6 after commissioning the first cell are crucial (see Figure 4.1). The treatment system should be in operation near the beginning of Year 7. The reason the schedule is tight is that reasonable quantity and quality data will not be generated until sometime in Year 4. Alternatively, if data is available from another regional landfill, which operates in a similar fashion, such data may be useful for design.

Leachate containment may be more feasible than trucking. Since all preliminary designs presented in this report include recirculation, and the designs either include equalization or aeration basins, this part of the treatment system infrastructure could be put into place early in the life of the landfill. Odours could be controlled by aerating the equalization basin.

Leachate could also be trucked or pumped to the nearest municipal sewer. Expected daily flows should not exceed 15 m^3 until Year 5. After Year 5, the daily flow is dependent on the capping schedule. If cells are not capped, flows will increase dramatically and neither trucking nor containment will suffice. Lesser quantities of leachate will be produced if cells are capped upon, or soon after completion. In either case, however, these leachate management options are short term, and should no longer be considered viable approximately 7 years after start-up of the landfill.

It should be noted that the time frame given is for a landfill receiving less than 10 percent of its waste as municipal sludge. If municipal sludge represents a significant amount of the waste, field capacity will be reached faster. Therefore, treatment plant design and construction should be carried out earlier. Exactly how much earlier is dependent on the ratio of sludge to waste and the water content of the sludge.

3.5 Required Operating Lifetime of the Treatment Plant

A landfill leachate treatment plant, although not really required until some time after the start of the landfill, will be required long after the landfill itself is decommissioned. Expected leachate flows and strength after landfill completion

will vary depending on the capping strategy and whether or not leachate recirculation is practised. A projection of 45 years of leachate characteristics for different capping strategies carried out for this study and is summarized in Table 4.1. Projected Year 45 leachate BOD varies from 20 mg/L, for no cap, to 1400 mg/L for capping 5 years after all completion. Not mentioned in the table, but equally important as BOD for direct discharge, is the ammonia concentration in the wastewater. Ammonia will generally increase with the age of the landfill and may be as high as 200 mg/L in Year 45 for all three capping strategies. Also not included in the table is the expected metal ion concentrations which will also be high beyond Year 45. Both high metal ion and ammonia concentrations would, even with low organic strength, prevent direct discharge of the leachate to the receiving water.

Because of the long service lifetime of a leachate treatment plant, certain components of the plant will have to be replaced in addition to regular maintenance. The structures and civil works should be serviceable for approximately 40 years. Mechanical and electrical components should have a service life of 20 to 25 years, so that replacement of all mechanical/electrical equipment should be expected during the service life of the plant.

Naturally, it is difficult to predict when this recapitalization will need to occur as economics, new effluent restrictions and technological advances may dictate the timing as much as the state of the original treatment plant.

4.0 PROJECTION OF LEACHATE QUANTITY AND QUALITY

4.1 Introduction to the HELP Model

The Hydrologic Evaluation of Landfill Performance (HELP) computer model was developed by the U.S. Army Corps of Engineer as a management tool in estimating landfill leachate production. The HELP model is a quasi-two-dimensional hydraulic model of water movement into, across, through and out of landfills. Both main frame and personal computer versions of the model are in existence.

The model accepts meteorological, soil and landfill design data. Meteorological data may include daily precipitation, mean monthly temperatures, the length of growing season, insolation, leaf area index and winter cover factors. It is possible to use, in full or in part, the default meteorological data which is archived, on disc, for over 100 American cities. Soil data inputs are porosity, field capacity, hydraulic conductivity and initial water content. Default soil data (18 soil types) may be selected, or soil data may be user-specified. And finally, the landfill design data includes landfill area, depths for each layer and slope of the drainage layers. All landfill design data must be specified by the user.

The HELP model was developed to facilitate the assessment of the affects of landfill design on leachate quantity. In this respect, the HELP model was not made to accurately predict leachate volume. However, in a comparative sense, the model functions very well. In addition, for preliminary wastewater treatment design, outputs from the HELP model are useful in preliminary design and costing.

When the landfill design, soil and meteorological data are put into the model, the model output will include calculations of evapotranspiration, runoff, percolation through each drainage layer, hydraulic head on each barrier layer, seepage through the bottom of the landfill and the change in water storage of each layer during the prescribed time. Up to ten years of output can be simulated from any given input condition; the output can be as specific as daily values, but monthly and yearly averages may be more appropriate. Output for periods greater than 10 years can be obtained by altering input data to reflect the conditions present at the end of the previous simulation.

Landfills are composed of functionally different layers. Three layer types are recognized by the model: the barrier layer, which inhibits vertical percolation of water/leachate; the vertical percolation layer (eg., the waste layer), which allows vertical flow of water/leachate; and the lateral drainage layer, which provides collection of either surface water or leachate.

Typical completed landfills are composed of a combination of the layers described above. Normally from bottom to top there are a barrier layer, leachate drainage and collection layer, waste layer, prewaste barrier layer, lateral surface drainage and runoff collection layer, and finally, the cover or cap. The HELP model can simulate up to 12 layers, four of which may be barrier layers.

Landfills are generally divided into subsections or cells. Usually, individual cells are filled in sequence, but may or may not be capped upon completion. Therefore, operational landfills are composed of cells in various stages. One limitation of the HELP model is that any one stage must be modelled for a complete year, even if that stage exists for a shorter period of time.

How the landfill is operated has dramatic bearing on the volume of leachate to be expected. Even for well operated landfills of equal depth and area, where groundwater infiltration and surface run-on are minimized, leachate flow can vary by an order of magnitude based on when, and if, completed cells are capped. For the purposes of this report, two cases, which represent the extremes of leachate quantity, were simulated. In Case 1, individual cells are capped 5 years after cell completion; in Case 2, completed cells are not capped until closure of the landfill. Results of these simulations are reported in Section 4.3.

4.2 Assumptions Made When Using the HELP Model

There are many technical factors, some of which must be assumed for those landfills not yet constructed, which aid the HELP model to determine leachate quantity. A brief description of each factor, along with the assumptions made, will follow.

Field Capacity

Waste entering landfills is usually not saturated with water. Of course, the extent of saturation is dependent on the waste source. As water enters the landfill, it is soaked up by the waste until field capacity is attained. Until field capacity is met, little leachate is produced; after it is met, there is approximately a one-to-one relationship between moisture entering the waste layer and leachate quantity. The waste mixture in this study was assumed to have a 20 percent (vol/vol) moisture content with an assumed field capacity of 33 percent.

Cover Permeability

Uncovered landfills in the maritimes will convert approximately fifty percent of precipitation into leachate. Correctly engineered caps or covers can reduce leachate generation by an order of magnitude. The capped cells in the proposed Westmorland/Albert (WA) landfill will have a cap, composed of a vegetative layer on top of 1 metre of clay soil with an overall porosity of 10^{-6} cm/sec. Uncapped cells were modeled to have 30 cm of cover with no vegetation.

Configuration of the Landfill

A lesser quantity of leachate will be produced for equally engineered landfills when the depth to surface area ratio is increased. Therefore, newly planned landfills are being designed with greater waste layer depths. The WA landfill design waste layer depth is 15 metres.

Recycle

Recirculating leachate (to the landfill) serves several purposes: by recirculating the field capacity is attained much faster than with no recycle, which, in turn, promotes an increased rate of waste stabilization; recirculation of leachate promotes the use of the entire landfill as an equalization basin, thereby helping maintain a more constant waste stream strength; and finally, some degree of leachate treatment is achieved in landfills with recycle. Recirculation will not, however, in a climate similar to that experienced in the maritime provinces, serve to lessen leachate production. Recirculation will also result in additional pumping and distribution costs.

It is generally agreed that the advantages of recirculation outweigh the additional pumping costs, and therefore, recirculation of leachate is included in the preliminary treatment system designs presented in this report.

Precipitation

The HELP model has a utility which allows up to 20 years of monthly precipitation data to be generated from historical daily records of rainfall and temperature. In addition, there is another utility which allows transformation of the archived climatological data, which exists for over 100 cities in the United States, with respect to the average monthly temperature and precipitation data for the city of choice in the same geographic proximity. For the purpose of this study, the average data for Moncton was used to transform existing data for Caribou, Maine. In doing so, the rainfall pattern calculated for Moncton may be slightly different than the actual but average yearly precipitation will be equal to that of Moncton.

Evapotranspiration

Evapotranspiration is a function of the energy available (temperature and solar insolation), vegetative cover, evaporation zone depth and soil water content. Open landfills were assumed to have no vegetative cover and an evaporation depth of twenty centimetres; capped landfills were assumed to the covered with grass with an evaporation depth of fifty-one centimetres.

Runoff - run-on - infiltration

Stormwater management and impermeable bottom layer design are crucial to both minimizing leachate and preventing contamination of the surrounding area. In compliance with New Brunswick landfill management guidelines, the proposed landfill will have an impermeable bottom layer which will minimize infiltration.

Run-on should never occur for properly managed landfills and was considered to be zero at all times. No surface runoff from active cells is to be allowed, and therefore, all precipitation, less evapotranspiration will become leachate. For capped cells, the runoff and most of the cap drainage layer water will be collected and will not be considered as leachate.

Cap Infiltration and Leachate Collection Systems

Both drainage systems were specified as a 46 cm deep soil with a hydraulic conductivity of 1×10^{-2} cm/sec. Drainage pipes were spaced at 55 m on a two percent slope.

Landfill Design and Loadings

The landfill will be designed to accept 86,000 tonnes of waste per year over 25 years. The average waste/soil mixture was assumed to be 700 kg/tonne with water content of 20 percent vol/vol. At this loading, and with a waste to soil mixture of 4:1, (wt.wt) approximately 0.9 hectares will be needed per year, for a total landfill area of approximately 23 hectares (25 cells) for the 25 year active lifetime.

Capping of Completed Cells

Three capping management scenarios will be investigated: the first scenario is to cap active cells 5 years after completion; the second scenario is to cap all landfill cells after closure of the landfill (i.e., after 25 years); and the third with no capping at all. The period of 5 years was chosen so as to allow the waste layer to reach field capacity prior to capping.

Year One Leachate Production

The method of evaluating incomplete cell leachate production, put forth in "The Assessment of Leachate Generation and Treatment Options" report by Owen Washburn Associates Ltd., February, 1990, was used in this study. In summary, the active cell was divided into quarters and the model was run for one year with ¼, ½ and ¾ of the cell empty. Any precipitation that fell in the empty part of the cell was considered to be potential leachate since the drainage system for the entire cell was also assumed to be in place at the beginning of the year.

4.3 Projected Leachate Quantity using the HELP Model

The HELP model was used to predict leachate quantity for the proposed Westmorland/Albert landfill under the following conditions:

- individual cells are capped 5 years after becoming inactive;
- all cells are capped upon completion of the landfill (25 years); and
- no cap (final cover) on any cell.

The model was run using transformed climatological data from Caribou, Maine. The transformation was carried out within the model and was dependent on the average monthly precipitation and temperature, as well as growing season, for Moncton, N.B.

In addition to the assumptions listed in the previous section, the following procedure was followed to facilitate modelling:

- there was no hydraulic connection between cells; and
- the amount of precipitation that fell on any 1 cell was dependent only on how long the cell was in service. Cell 2 would receive the same precipitation as Cell 1 did the previous year.

Figure 4.1 displays daily leachate production for each of the three cases cited. Maximum daily flows varied from 36 to 320 m^3/d for the cases of being capped after 5 years, and no cap, respectively.

4.4 Projected Leachate Quality Characteristics and Loadings

Leachate Quality

Leachate quality modelling is summarized in the previously mentioned Owen Washburn Associates Ltd. report. All existing models require major assumptions to be made. The highly variable nature of leachate characteristics make assuming leachate qualities extremely difficult. Since the available simplistic models' outputs are highly sensitive to the assumptions made, the value of these models in predicting leachate quality is questionable.

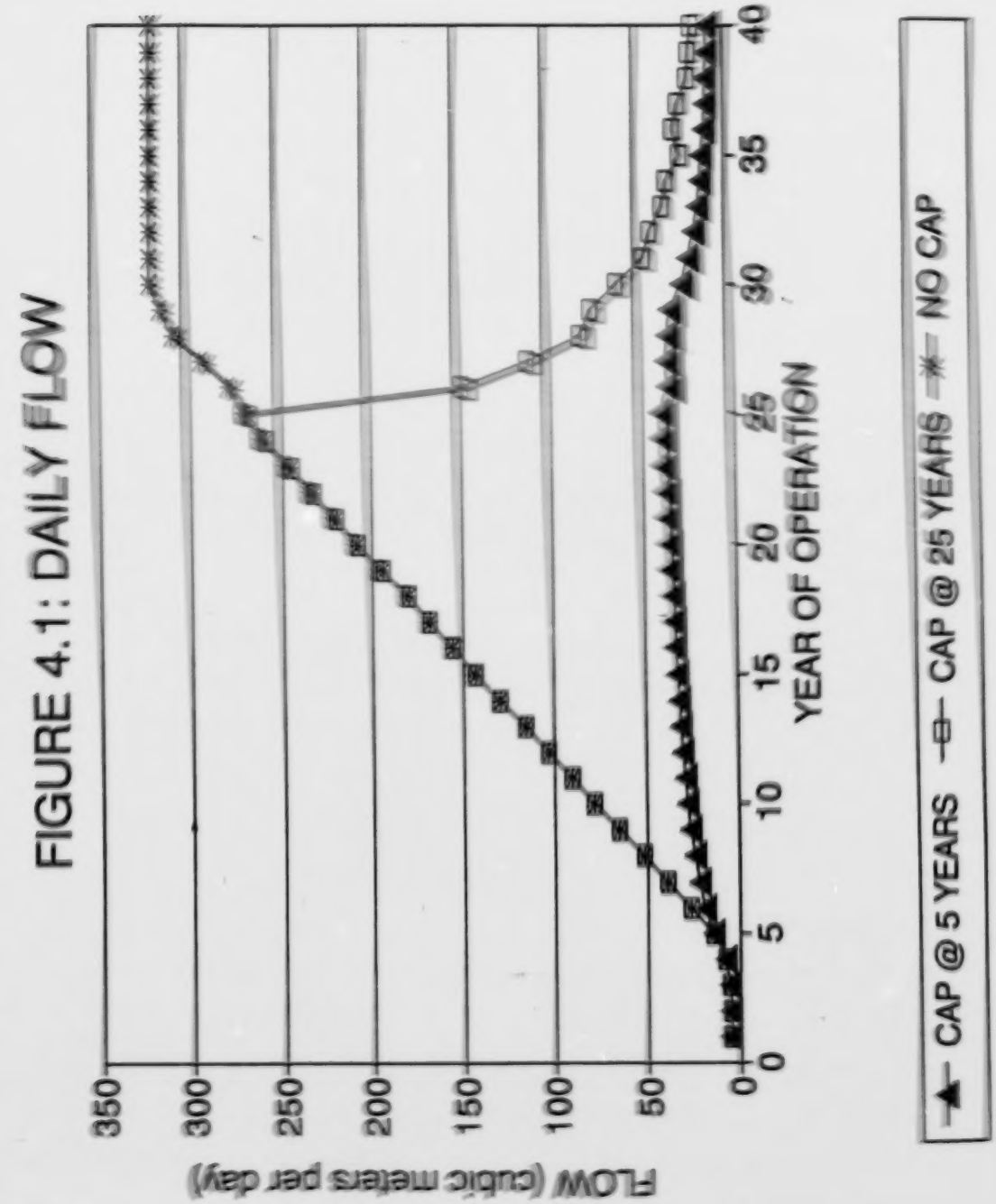
FIGURE 4.1: DAILY FLOW
FLOW (cubic meters per day)
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
YEAR OF OPERATION
CAP @ 5 YEARS
CAP @ 25 YEARS
NO CAP

The generally accepted facts about leachate characteristics are:

- leachate strength is strongest in younger landfill cells (2 - 8 years after completion);

- there is a correlation between the strength of leachate and the amount of moisture that has passed through the waste; and

- prior to attainment of field capacity, weak leachate is produced, but the strength increases until field capacity is attained because the average hydraulic retention time is longest at this point.

Based on these accepted facts, and actual leachate quality from an existing landfill (Fredericton), estimations can be made as to the strength of leachate to be expected from the three landfill operation cases.

For the case where cells are capped after 5 years and recycle is practised, landfill leachate BOD should reach its maximum at approximately the same time as daily flows peak. It is expected that the maximum leachate BOD would be approximately 12,000 mg/L. Leachate composition should be relatively steady-state for much of the life of landfills operated in this manner.

For the cases where cells are capped either not at all, or at the end of the active period of the landfill (25 years), the expected BOD strength would be different. Cells will reach saturation much quicker in these cases due to both more leachate production and recycle. The effect of quick saturation of the landfill cell will be a faster rate of stabilization. Therefore, BOD would be expected to peak much

earlier and higher for each cell and then drop off exponentially thereafter. A peak cell BOD in the order of 25,000 mg/L would be expected. However, the cell which produces a leachate strength of 25,000 mg BOD/L would be attenuated by the leachate from other cells whose BOD might be higher or lower depending on when they came into service. Therefore, the peak landfill BOD would not be 25,000 mg/L. A mathematical estimation of peak BOD, based on equal flow from each cell (after saturation), equal strength from each cell of equal age, and an exponential decrease in strength after the peak is achieved, suggests that a peak BOD of near 17,000 mg/L would occur approximately 7 years after start-up of the landfill.

BOD Loadings

Two of the important quantities needed for successful treatment plant design are peak flow and peak loading. With flow equalization, peak flow will be, for the purposes of this report, the average maximum flow expected. Peak load is a little more complicated to arrive at than peak flow in that the maximum load is dependent on the product of flow and concentration. Since periods of maximum flow may have very low concentrations of BOD, loading during peak flow may or may not be the peak loading.

For the case when cells are capped after 5 years, peak load will occur at the same time as peak flow. As a result the peak load is simply a function of the product of peak flow and peak concentration. In this case, a peak flow of 36 m^3/d with 12,000 mg/L BOD results in a peak load of 440 kg/d of BOD.

For the case of no capping until the landfill is complete, peak flow does not occur until the last operating year of the landfill, year 25 (or 5 years thereafter if no capping at all is done). However, as was previously mentioned, the peak concentration of BOD should occur approximately on Year 7, after which BOD concentration will decrease. The rate of decrease is difficult to predict, but based on the same assumptions used to calculate the maximum landfill leachate concentration, the concentrations for the Years 1 through Year "n" can be calculated. Using daily flow predictions along with concentrations, the loads can be estimated. Predicted daily flows and concentration, along with calculated loads are presented in Table 4.2. From this model the peak BOD concentration occurs in Year 7 (16,800 mg/L) where as the peak BOD loading occurs during Years 25 at 1100 kg/d.

The design flows and loadings for all three landfill operational cases are summarized in Table 4.1.

TABLE 4.1
ESTIMATED DESIGN FLOW AND MAXIMUM BOD LOADING

	CAPPING AFTER	MAXIMUM FLOW (m^3/d)	MAXIMUM BOD (mg/L)	PEAK LOAD (kg BOD/d)
Case 1	5 years	36	12000	440
Case 2	25 years	270	16000	1100
Case 3	no cap	320	16000	1100

TABLE 4.2 - PROJECTED LEACHATE FLOWS AND BOD LOADINGS

CAPPING STRATEGIES									
	Cap @ 5 Years			Cap @ 25 Years			No Cap		
Year	Flow (M3/d)	BOD (mg/l)	Load (kg BOD/d)	Flow (M3/d)	BOD (mg/l)	Load (kg BOD/d)	Flow (M3/d)	BOD (mg/l)	Load (kg BOD/d)
1	5	500	2	5	500	2	5	500	2
2	5	500	2	5	500	2	5	500	2
3	5	1700	9	5	1700	9	5	1680	9
4	8	4500	34	8	4500	34	8	4510	34
5	14	8600	123	14	8600	123	14	8560	123
6	19	9800	181	26	12800	338	26	12840	338
7	22	10600	231	39	16800	653	39	16770	653
8	24	11100	269	51	16300	835	51	16350	835
9	26	11600	302	65	15000	977	65	14970	977
10	27	11900	327	79	13100	1031	79	13060	1031
11	29	12000	345	91	11400	1039	91	11430	1039
12	30	12100	358	103	10100	1044	103	10090	1044
13	31	12100	369	116	9100	1047	116	9060	1047
14	31	12000	377	130	8100	1051	130	8100	1051
15	32	12000	385	144	7300	1054	144	7350	1054
16	32	12000	391	155	6800	1057	155	6800	1057
17	33	12000	396	168	6300	1059	168	6300	1059
18	34	11900	400	180	5900	1062	180	5890	1062
19	34	11900	404	194	5500	1064	194	5480	1064
20	34	11800	408	208	5100	1067	208	5130	1067
21	35	11800	411	220	4900	1068	220	4860	1068
22	35	11700	414	223	4600	1170	233	4600	1070
23	36	11700	416	245	4400	1071	345	4380	1071
24	36	11600	417	259	4100	1073	259	4140	1073
25	36	11500	418	267	4000	1074	267	3950	1074
26	32	13100	416	145	7400	1072	275	3840	1073
27	32	13000	417	108	9900	1072	290	3860	1074
28	32	12900	410	80	13400	1065	305	3510	1068
29	30	12900	386	74	14100	1041	312	3310	1044
30	23	12700	297	60	15900	952	318	2980	955
31	20	12200	239	44	16800	737	318	2300	741
32	17	11400	189	40	10500	422	318	1320	427
33	15	10400	151	33	7190	240	318	760	244
34	13	9120	119	32	3130	99	318	320	103
35	12	7820	93	24	1810	44	318	150	48
36	11	6820	75	28	1310	37	318	130	41
37	10	6020	62	25	1290	32	318	110	36
38	10	5290	52	19	1450	28	318	100	33
39	9	4650	43	19	1290	25	318	90	29
40	9	3990	36	17	1210	21	318	80	26
41	8	3770	30	17	1090	19	318	70	23
42	8	3080	25	17	1200	16	318	70	21
43	8	2530	20	16	1020	14	318	60	19
44	8	2030	16	16	900	11	318	50	16
45	8	1560	12	15	810	9	318	40	14

5.0 PRELIMINARY LEACHATE TREATMENT DESIGNS

5.1 Introduction

There are two recommended leachate treatment options: modified aerated pretreatment lagoon; and a complete treatment option composed of physical chemical treatment, followed by anaerobic and aerobic treatment in sequence. The aerated lagoon option would only be selected if the treated effluent is to proceed to a municipal sewer; the second option would be selected if complete leachate treatment is needed before discharge to a receiving water.

The design of any leachate treatment system is dependent on how the landfill is managed with respect to the schedule of cell capping, three cases of which were presented in Section 4.0. Since two of the landfill management cases were similar with respect to flow and loadings, only the cases where individual cells are capped after 5 years and where all cells are capped after 25 years will receive preliminary design and costing.

For comparison purposes, both cases will be designed and costed for both treatment options, resulting in four preliminary designed and costed systems.

5.2 Aerobic Pretreatment

Multi-cell aerated lagoons can be an effective pretreatment option if a municipal sewer system is large enough to accept the flow. Two lagoon systems will be designed for the landfill management Cases 1 and 2.

It is a fact that secondary treatment systems will operate more efficiently at higher temperatures. By covering and insulating one or more cells of a lagoon system, it can be made to operate at a warmer temperature during winter months, which will result in a better effluent quality.

The two aerated lagoon systems will have the following in common:

- 4 equal-volume cells with baffled intra-cell divisions;
- surface self-aspirating type aerators;
- option to cover the cells;
- quiescent zone in final cell for solids settling; and
- designed to meet limits of 600 mg/L of BOD and 500 mg/L of SS.

Case 1: Lagoon System Design - Cells Capped After 5 Years

The design flow and loadings are 40 m^3/d and 450 kg BOD/d, respectively. To meet the New Brunswick Guidelines for hydraulic retention time (HRT) at winter operating temperatures, a total lagoon volume of 1600 m^3 divided into 4 equal cells would be necessary. The required aeration would be 50 hp, divided into 20:20:5:5 for cells 1 through 4, respectively.

The lagoon system should easily meet the 600 mg BOD/L and 500 mg SS/L limit. If metal concentrations in lagoon effluent were too high then a lime addition system could be used to improve metals precipitation. Also, extra aeration would be necessary if conversion of ammonia to nitrate was required.

Case 2: Lagoon System Design - All Landfill Cells Capped After 25 Years

The design flow and loadings are 320 m^3/d and 1150 kg BOD/d, respectively. The necessary total lagoon volume would be 12000 m^3 divided into 4 equal cells, each having a volume of 3000 m^3. The required aeration would be 140 hp, divided into 80:40:10:10 hp for cells 1 through 4, respectively.

5.3 Complete Secondary Treatment

5.3.1 Introduction

The purpose of a complete on-site treatment system is to treat leachate to produce an effluent of sufficient quality to be discharged into a receiving water body. To treat leachate to this extent generally requires a combination of physical chemical and secondary treatment. The proposed secondary treatment train, following physical chemical treatment, is an anaerobic reactor followed by aerobic sequencing batch reactor polishing. A schematic of the complete system is given in Figure 5.1 and a brief description of the unit process design will follow in this chapter. System costs are provided in Section 6.0.

5.3.2 Physical Chemical Pretreatment

Since the advantages of physical chemical (PC) pretreatment, outweigh post-secondary PC treatment, the PC pretreatment option alone is presented.

The PC pretreatment system will operate in the flow through mode and will consist of the following:

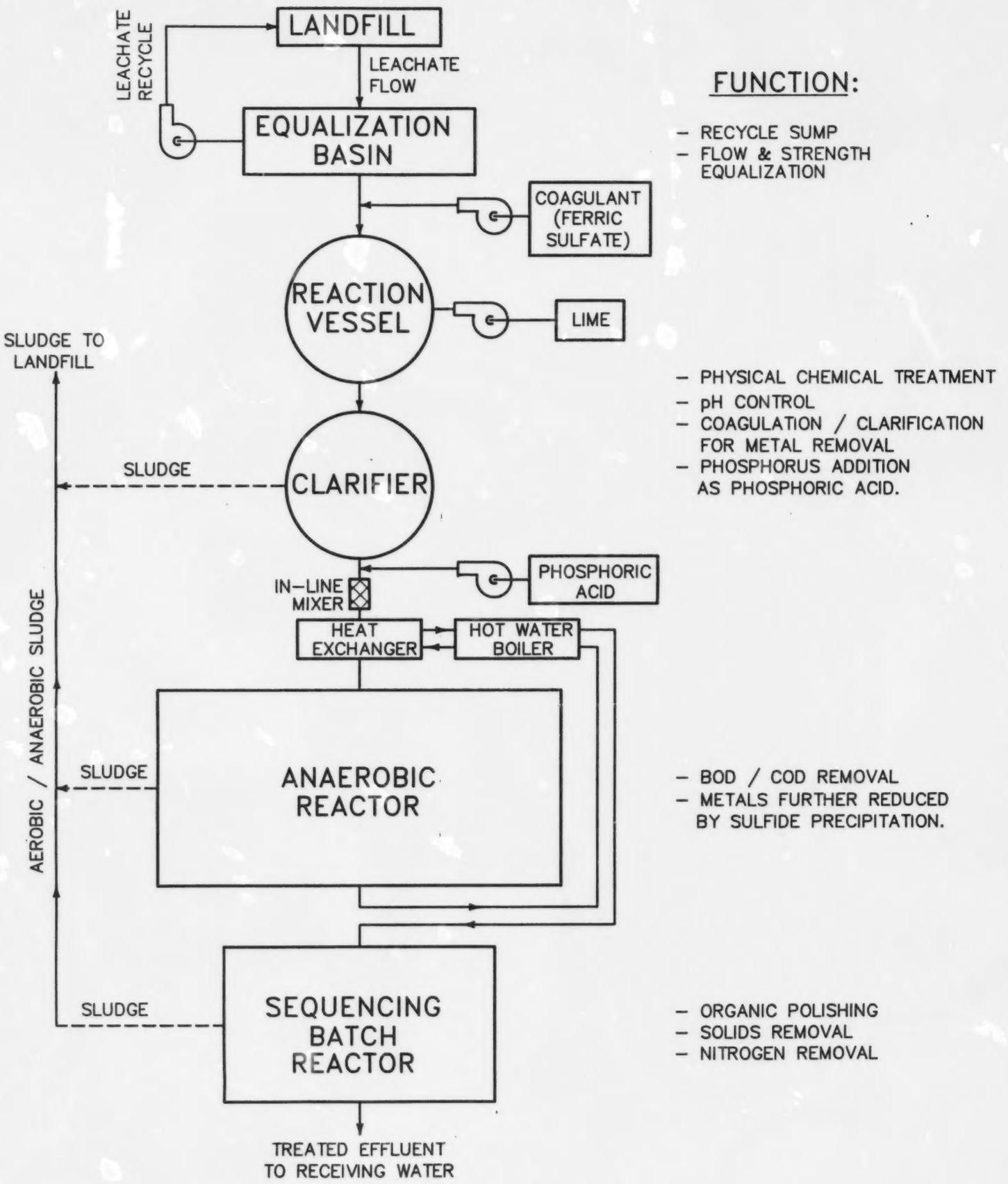

FIGURE 5.1
COMPLETE LEACHATE TREATMENT SYSTEM

- a reaction vessel;
- a reaction vessel mixer;
- a ferric sulfate feed system;
- a lime (or caustic) feed/storage system;
- a flocculating clarifier;
- duplex sludge pumps;
- pH control dosing equipment;
- an in-line mixer; and
- a pH control system.

The purposes of pretreatment are to precipitate metal ions in metal-hydroxide complexes by raising the pH of the leachate, and following metal ion removal, to readjust the pH (as necessary) for subsequent biological treatment. Phosphoric acid may be used for pH control, as well as to provide the phosphate nutrient.

There may be some high molecular weight organic matter removal through the PC system (typically 10% of total COD) but for preliminary design purposes this will be ignored.

Each leachate management case requires a pretreatment system which size is based on the flow expected.

5.3.3 Anaerobic and Aerobic Polishing

Anaerobic treatment followed by an SBR system for organic and nitrogen polishing has promise to be the most cost effective secondary treatment method for

producing an on-site high quality effluent. Designs for the two cases are presented in the paragraphs to follow.

Case 1: Anaerobic/Aerobic Design - Cells Capped After 5 Years

Anaerobic Reactor

With a design and loading of 40 m^3/d and 450 kg BOD/d (900 kg COD/d), an anaerobic reactor volume of 1000 m^3 would be required giving a design HRT of 25 days. The reactor would be an insulated steel tank with an epoxy coating on the inside. The tank would have a cover system made of Seaman, XR-5 flexible membrane and 2 inches of closed-cell foamed polyethylene insulation; the outside tank wall insulation would be 2 inches of sprayed on foam.

The anaerobic reactor would be equipped with the following:

- four gas liquid solids separators;
- sludge recycle/wasting pumps;
- a slow speed mixer; and
- biogas handling system.

The anaerobic reactor should operate efficiently at temperatures of 20°C, and above. Influent to the anaerobic system would be preheated in a heat exchanger by a hot-water boiler. Biogas generated in the reactor could be used to help maintain design temperatures. The use of landfill methane generation for additional heating potential should be investigated.

Sequencing Batch Reactor

The SBR system would be two concrete basins with a total HRT of 2 days. The resultant total basin volume would be 80 m^3. The aeration system design would be based on a loading of 100 kg BOD/d (80 percent BOD removal in the anaerobic reactor) and 10 kg of ammonia nitrogen per day. The required equipment for the SBR would be:

- two inground concrete basins (40 m^3 each);
- submerged aeration system (40 coarse bubble diffusers);
- duplex blowers with 20 hp each capable of delivering 200 cfm each at a pressure of 11 psi;
- feed and decant pump systems;
- waste sludge pumps;
- two 5 hp mixers;
- Programmable Logic Controller; and
- control building for both anaerobic and aerobic systems.

The effluent from the SBR should be of high enough quality to meet the effluent guidelines of 20 mg/L BOD and SS. In addition, the SBR should be capable of near complete ammonia transformation (200 mg/L → <20 mg/L). One important feature is the flexibility of the SBR which allows, if desired, nitrogen removal (50 percent) with minor operational changes, by establishing anoxic periods in the cycle to promote denitrification.

Case 2: Anaerobic/Aerobic Design - All Landfill Cells Capped After 25 Years

Anaerobic Reactor

The design flow and loadings to the anaerobic reactor would be 320 m^3/d and 1150 kg BOD/d (2200 kg COD/d). The required reactor volume is 2200 m^3 giving a design minimum HRT of approximately 7 days. The anaerobic system is similar to that recommended in the previous case (i.e., tank construction, 2" insulation) but with mechanical components resized accordingly.

Sequencing Batch Reactor

The SBR system volume, allowing for 2 days HRT, would be 640 m^3. As with the previous case, the basins would be inground, concrete construction.

Allowing for 80 percent BOD removal in the anaerobic reactor, the SBR aeration system would be sized for 230 kg BOD/d and 10 kg ammonia nitrogen/d. The required equipment for the system is as follows:

- two inground concrete basins (320 m^3 each)
- submerged aeration system (75 coarse bubble diffusers);
- duplex blowers with 30 hp each capable of delivery 360 cfm each at a pressure of 11 psi;
- feed and decant pump systems;
- waste sludge pumps;
- two-5 hp mixers;

- PLC; and
- control building.

6.0 COSTS

6.1 Introduction

The objective of this section is to summarize the estimated capital and annual costs for aerobic pretreatment and complete secondary treatment for both Cases 1 and 2 (as discussed previously in Section 5.0). These are not detailed capital costs because specific site information is not available. Estimated costs do not include connections to a municipal sewer system or outfalls to the nearest watercourse, since location of the leachate WWTP was not fixed. However, based upon preliminary estimates, connection to the municipal sewer system would cost approximately $700,000 to $1,000,000. In addition, costs for the leachate collection and recycle systems are not included.

6.2 Aerobic Pretreatment

Table 6.1 presents an estimated capital cost summary for the aerobic pretreatment option for both Cases 1 and 2.

TABLE 6.1

AEROBIC PRETREATMENT CAPITAL COSTS

ITEM	ESTIMATED COST	
	CASE 1 (Capped After 5 Years)	CASE 2 (Capped After 25 Years)
Earthwork	$ 60,000	$ 150,000
Membrane Liner and Baffles	$100,000	$ 250,000
Aerators	$120,000	$ 220,000
Yard Piping	$100,000	$ 150,000
Control Building	$ 50,000	$ 50,000
Electrical	$ 50,000	$ 100,000
Contingencies & Engineering	$120,000	$ 230,000
GST	$ 40,000	$ 80,000
TOTAL	$640,000	$1,230,000

6.3 Complete Secondary Treatment

Table 6.2 presents as estimated capital cost summary for the complete secondary treatment option for both Cases 1 and 2.

TABLE 6.2
COMPLETE SECONDARY TREATMENT CAPITAL COSTS

ITEM	ESTIMATED COST	
	CASE 1 (Capped After 5 Years)	CASE 2 (Capped After 25 Years)
Equalization Basin	$ 200,000	$ 300,000
Pumps to Physical/ Chemical Treatment	$ 60,000	$ 100,000
Coagulant Storage and Feed System	$ 60,000	$ 90,000
Mixed Reaction Vessel	$ 30,000	$ 50,000
Lime Storage and Feed Systems	$ 200,000	$ 250,000
pH Control System	$ 30,000	$ 30,000
Clarifier	$ 140,000	$ 220,000
Sludge Pumps	$ 20,000	$ 40,000
Phosphoric Acid Storage and Feed Pumps	$ 40,000	$ 60,000
Anaerobic Reactor	$1,000,000	$1,400,000
Heat Exchanger/ Hot Water Boiler	$ 200,000	$ 250,000
SBR System	$ 450,000	$ 700,000
Yard Piping	$ 250,000	$ 350,000
Electrical	$ 200,000	$ 250,000
Instrumentation and Controls	$ 150,000	$ 200,000
Control Building	$ 150,000	$ 200,000
Contingencies & Engineering	800,000	$1,100,000
GST	$ 270,000	$ 390,000
TOTAL	$4,260,000	$5,980,000

6.4 Estimated Annual Costs

The estimated annual costs are summarized in Table 6.3. More complete information, including the assumption made, is provided in Appendix A.

TABLE 6.3
ESTIMATED ANNUAL COSTS

Capping Strategy	Treatment Option	Labour	Electrical	Maintenance	Supplies (including fuel) and Laboratory Testing	Total
Cells Capped After 5 Years	Pretreatment	15,000	20,000	17,000	20,000	72,000
	Complete	90,000	14,000	113,000	51,000	268,000
Cells Capped After 25 Years	Pretreatment	15,000	55,000	31,000	20,000	121,000
	Complete	90,000	18,000	153,000	268,000	529,000

APPENDIX A

ANNUAL COSTS

ANNUAL COSTS

Estimates of the annual operation and maintenance costs are presented for both pretreatment systems and both complete treatment systems, including:

- labour;
- electrical;
- maintenance; and
- supplies and laboratory testing.

The following assumptions were made in completing the estimates:

- labour at $30,000 per man-year;
- electricity at $0.05 per kWh; and
- maintenance costs proportioned at 1.5 percent per civil works and 5 percent per mechanical works based on capital cost.

Labour

Pretreatment

- assume ½ shift for miscellaneous operations
- (½) ($30,000) = $15,000/a

Complete Treatment

- Assume 3 shift positions for operations
- (3) ($30,000) - $90,000/a

Electrical

Pretreatment

Cap After 5 Years:

- Aerators - 50 hp
- Power factor = 0.9

Cost = (50 hp (0.9) (24 h/d) 365 d/a) ($0.05/kWh)

= $20,000/a

Cap After 25 Years:

- Aerators - 140 hp

Cost = (140 hp) (0.9) (24 h/d) (365 d/a) ($0.05/kWh)

= $55,000/a

Complete Treatment

Cap After 5 Years:

- Aeration - 20 hp
- Mixing - 5 hp
- BVF Mixer - 5 hp
- Sludge Pumps - 5 hp

Total - 35 hp

Cost = (35 hp) (0.9) (24 h/d) (365 d/a) ($0.05 (kWh)

= $14,000/a

Cap After 25 Years:

- Aeration - 30 hp
- Mixing - 5 hp
- BVF Mixer - 5 hp
- Sludge Pumps - 5 hp

Total - 45 hp

Cost = (45 ph) (24 h/d) (365 d/a) ($0.05/kWh)

= $18,000/a

Maintenance

Pretreatment

Cap After 5 Years:

Total Mechanical = $200,000

Total Civil = $440,000

Cost = (200,00) (0.05) + (440,000) (0.015)

= $17,000/a

Cap After 26 Years:

Total Mechanical = $350,000

Total Civil = $880,000

Cost = (350,00) (0.05) + (880,000) (0.015)

= $31,000/a

Complete Treatment

Cap After 5 Years:

Total Mechanical = $1,400,000

Total Civil = $2,860,000

Cost = (1,400,000) (0.05) + (2,860,000) (0.015)

= $113,000/a

Cap After 25 Years:

Total Mechanical = $1,800,000

Total Civil = $4,180,000

Cost = (1,800,000) (0.05) + (4,180,000) (0.015)

= $153,000/a

Supplies and Laboratory Testing

Pretreatment:

Caustic and Phosphoric Acid = $15,000/a

Lab Testing = $5,000/a

Total = $20,000/a

Complete Treatment:

	Cap After 5 Years	Cap After 25 Years
Lime	$ 10,000	$ 80,000
Ferric Sulfate	$ 6,000	$ 48,000
Phosphoric Acid	$ 5,000	$ 40,000
Lab Testing	$ 20,000	$ 20,000
Total	$ 41,000/a	$188,000/a

Fuel To Preheat Wastewater

Assume
- Leachate Temperature = 5°C
- Heat to = 22°C
- Temperature change = 17°C - 31°F
- Leachate flow = 40 m^3/d = 10,560 USgpd
- Total transfer efficiency = 70%
- Energy cost = \$7.00/10^6 BTU

Cost = (10,560 USgpd) (8.34 lb/gal) (31°F) (365 d/a) (1/0.7) (1/10^6)
(\$7.00/10^6 BTU)
= \$10,000/a

Similarly, for 370 m^3/d (cap after 25 years), the full cost would be \$80,000/a.